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Verification of a Simplified Method to Evaluate the Capacities of Template-Type Platforms

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ABSTRACT

This paper summarizes development of simplified procedures to evaluate storm loadings imposed on template-type platforms and to evaluate the ultimate limit state lateral loading capacities of such platforms. Verification of these procedures has been accomplished by comparing results from the simplified analyses with results from three-dimensional, linear and nonlinear analyses of a variety of template-type platforms. Good agreement between results from the two types of analyses has been developed for the evaluations of both loadings and capacities.

The verification platforms have included four-leg well protector and quarters structures and eight-leg drilling and production Gulf of Mexico structures that employed a variety of types of bracing patterns and joints. Several of these structures were subjected to intense hurricane storm loadings during hurricanes Andrew, Carmen, and Frederic. Within the population of verification platforms are several that failed or were very near failure. The simplified loading and capacity analyses are able to replicate the observed performance of these platforms. Realistic simulation of the brace joints and foundation capacity charcteristics are critical aspects of these analyses. There is a reasonable degree of verification of the simplified methods with the observed performance of platforms in the field during intense hurricane storm loadings.

These methods can be used to help screen platforms that are being evaluated for extended service. In addition, the results from these analyses can be used to help verify results from complex analytical models that are intended to determine the ultimate limit state loading capacities of platforms. Lastly, and perhaps most importantly this approach can be used in the preliminary design of new platforms.

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INTRODUCTION

During the past three decades, an immense amount of effort has been devoted to development of sophisticated computer programs to enable the assessment of storm wind, wave, and current loadings and the ultimate limit state capacity characteristics of conventional, pile-supported, template-type offshore platforms.[1-3] These programs require high degrees of expertise to operate properly, are expensive to purchase and maintain, and require large amounts of manpower and time to complete the analyses. Due to the sophistication of these programs, experience has shown that it is easy to make mistakes that are difficult to detect and that can have significant influences on the results.[4]

This paper summarizes the second phase of verification of simplified procedures to evaluate environmental loadings and ultimate limit state lateral loading capacities of template-type platforms. Reasonable simplifications and high degrees of "user friendliness" have been employed in development of the computer software to reduce the engineering effort, expertise, and costs associated with the analyses.

The computer program that has been developed to perform the simplified analyses has been identified as ULSLEA (Ultimate Limit State Limit Equilibrium Analyses).[5,6]

The first phase of development and verification of these procedures has been documented.[7,8] The first phase developments were verified with comparisons of observed and computed loadings and capacities from five 8-pile self-contained drilling and production platforms and one 5-pile well protector. The simplified static capacity bias (nonlinear analysis capacity / simplified analysis capacity) ranged from 0.80 to 1.07 with a mean value of 0.95. Comparisons of the computed lateral load capacities based on the simplified approach with the estimated maximum loadings sustained by these platforms during past hurricanes indicated good agreement.

During the second phase of this research, based on the experience from the first phase developments, a number of improvements were made in the simplified analyses. These improvements are detailed by Bea and Mortazavi.[6]

Verification of the second phase procedures is demonstrated in this paper with comparisons of the results from the advanced simplified analyses with the results from three dimensional, linear and nonlinear analyses of template-type platforms. As in the first phase, good agreement between results from the two type of analyses has been developed for the evaluations of capacities. The verification platforms include three 4-leg structures and two 8-leg drilling and production platforms. These Gulf of Mexico platforms employed a variety of types of bracing patterns and joints. Several of these platforms were subjected to intense hurricane storm loadings during hurricanes Andrew, Camille, and Frederic. Within the population of verification platforms are several that failed or were very near failure. The simplified loading and capacity analyses are able to replicate the general performance of these platforms. Details of the nonlinear analyses of the second phase verification platforms have been documented.[4]

SUMMARY OF APPROACH

Using the concept of plastic hinge theory, limit equilibrium is formulated by implementing the principle of virtual work. This is the key to the simplified ultimate limit state analysis method. Where of importance, geometric and material nonlinearities are considered. This method is being increasingly used in plastic design of simple structures or structural elements (e.g. moment frames, continuous beams). Due to the impracticality of such analyses for more complicated structures, these methods have not found broad use in design or assessment of complex structures; all possible failure modes need be considered and evaluated to capture the "true" collapse mechanism and the associated ultimate lateral load.

Actual field experience and numerical results from three dimensional nonlinear analyses performed on a wide variety of template-type platforms indicate that in most cases certain failure modes govern the ultimate capacity of such platforms: plastic hinge formation in the deck legs and subsequent collapse of the deck portal, buckling of the main load carrying vertical diagonal braces in the jacket (and / or associated joint failures), lateral failure of the foundation piles due to plastic hinge formation in the piles and plastification of foundation soil, and pile pull-out or pile plunging due to exceedance of axial pile and soil capacities.

Within the framework of a simplified analysis and based on experience, collapse mechanisms are assumed for the three primary components that comprise a template-type platform: the deck legs, the jacket, and the pile foundation. Based on the presumed failure modes, the principle of virtual work is utilized to estimate the ultimate lateral capacity for each component and a profile of horizontal shear capacity of the platform is developed.

Storm intensity is based on the expected maximum wave height with wind speed and current velocities that have the same principal direction and occur at the same time as the maximum wave height. Comparison of the storm shear profile with the platform shear capacity profile identifies the "weak link" in the platform system. The base shear or total lateral loading at which the capacity of this weak link is exceeded defines the ultimate lateral capacity of the platform, R_U.

With these results, the Reserve Strength Ratio (RSR) can be determined as

$$RSR = \frac{R_U}{S_R} \qquad (1)$$

S_R denotes the reference storm total maximum lateral loading.

A computer program has been developed to perform the simplified analyses based on ULSLEA techniques.[5,6] High degrees of user friendliness have been employed in development of the software to reduce the engineering effort, required expertise, likelihood of errors, costs, and time associated with the analyses. User experience[4] with ULSLEA indicates that these attributes can be realized. Most importantly, the experience with ULSLEA indicates that its results can be used to determine the validity of results from more complex analyses. In every verification case cited herein, results from ULSLEA initially helped define major deficiencies and errors in either the complex analysis software or in the input to this software. Based on this experience, there is little doubt in the researchers' minds concerning the importance and utility of simplified methods.

Input Information

The geometry of the platform is defined by specifying a minimum amount of data by the user. These include the effective deck areas, the proportion and topology of jacket legs, braces, and joints, and of the foundation piles and conductors. The projected area characteristics of appurtenances such as boat landings, risers, and well conductors also must be specified. If marine fouling is present, the variation of the fouling thickness with depth may be specified by the user.

Specialized elements may be designated including grouted or ungrouted joints, braces, and legs. In addition, damaged (corrosion, holes, dents, bent, cracked) or defective elements (misalignments, under-driven piles) can be included. Dent depth and initial out-of-straightness are specified by user for braces with dents and global bending defects. User-defined element capacity reduction factors are introduced to account for other types of damage to joints, braces, and foundation elements.

Steel elastic modulus, yield strength, and effective buckling length factor for vertical diagonal braces are specified by the user. Soil characteristics are specified as the depth variation of "effective" undrained shear strength (for cohesive soils) or the "effective" internal angle of friction (for cohesionless soils). The effective soil characteristics are intended to recognize bias introduced by soil sampling, laboratory testing, and static analysis methods. A scour depth can be specified by the user.

Storm wind speed at the deck elevation, wave height and period, current velocity profile, and storm water depth are defined by the user. These values are assumed to be collinear and to be the values that occur at the same time. Generally, the load combination is chosen to be wind speed component and current component that occur at the same time and in the same principal direction as the expected maximum wave height. The wave period is generally taken to be expected period associated with the expected maximum wave height.

To calculate wind loadings acting on the exposed decks the user must specify the effective drag coefficient. Similarly, the user must specify the hydrodynamic drag coefficients for smooth and marine fouled members. User specified coefficients can also be introduced to recognize the effects of wave directional spreading and current blockage.

Environmental Loadings

Wave, current, wind, and storm tide are considered. Aerodynamic and hydrodynamic loadings are calculated according to API RP 2A guidelines.[9,10]

Wave horizontal velocities are based on Stokes 5th order theory. The specified variation of current velocities with depth is stretched to the wave crest and modified to recognize the effects of structure blockage on the currents. The total horizontal water velocities are taken as the sum of the wave horizontal velocities and the current velocities.

The maximum hydrodynamic force acting on the portions of structure below the wave crest are based on the fluid velocity pressure or drag component of the Morison Equation.

All of the structure elements are modeled as equivalent vertical cylinders that are located at the wave crest. Appurtenances (conductors, boat landings, risers) are modeled in a similar manner. For inclined members, the effective vertical projected area is determined by multiplying the product of member length and diameter by the cube of the cosine of its angle with the horizontal (to resolve horizontal velocities to normal to the member axis).

For wave crest elevations that reach the lower decks, the horizontal hydrodynamic forces acting on the lower decks are computed based on the projected area of the portions of the structure that would be able to withstand the high pressures.[11,12] The fluid velocities and pressures are calculated in the same manner as for the other submerged portions of the structure with the exception of the definition of the drag coefficient, *Cd*. In recognition of rectangular shapes of the structural members in the decks a higher *Cd* is taken. This value is assumed to be developed at a depth equal to two velocity heads (U^2/g) below the wave crest. In recognition of the near wave surface flow distortion effects, *Cd* is assumed to vary linearly from its value at two velocity heads below the wave crest to zero at the wave crest.[11]

Deck Leg Shear Capacity

The ultimate shear that can be resisted by an unbraced deck portal is estimated based on bending moment capacities of the tubular deck legs that support the upper decks.

A collapse mechanism in the deck bay would form by plastic yielding of the leg sections at the top and bottom of all of the deck legs. The interaction of bending moment and axial force is taken into account. The maximum bending moment and axial force that can be developed in a tubular deck leg is limited by local buckling of leg cross-sections.

The vertical dead loads of the decks are assumed to be equally shared between the deck legs. The vertical live loads in the deck legs caused by the lateral overturning forces are computed and summed to define the axial loading in each deck leg.

Due to relatively large axial loads (weight of the decks and topside facilities) and large relative displacements at collapse, P-Δ effects can play a role in reducing the lateral shear capacity and hence is taken into account.

To derive a realistic estimate of P-Δ effect with out leaving the framework of a simplified analysis, it is assumed that the deck is rigid. It is further assumed that plastic yielding of the sections at the bottom of the deck legs occur simultaneously, following the plastic yielding of the sections at the top of the legs and hence an estimate of plastic hinge rotations to calculate the deformations is unnecessary.

Finally, to estimate the deck bay drift at collapse Δ, the jacket is replaced by rotational springs at the bottom of each deck leg. The spring stiffness is approximated by applying external moments, which are equal in magnitude and have the same direction, to the top of jacket legs at the uppermost jacket bay. Assuming fixed boundary conditions at the bottom of these jacket legs, the rotation of cross-sections at the top of the legs and hence the rotational stiffness is determined.

The principle of virtual force is implemented to calculate the deck bay horizontal drift at collapse. Equilibrium is formulated using the principle of virtual displacement. Using the actual collapse mechanism as the virtually imposed displacement, the equilibrium equation for the lateral shear capacity of the unbraced deck portal is derived.

Jacket Bays Shear Capacity

The shear capacity of each of the bays of vertical bracing that comprise the jacket is estimated including the tensile and compressive capacity of the diagonal braces and the associated joint capacities. The capacity of a given brace is taken as the minimum of the capacity of the brace or the capacity of either its joints.

To derive a lower-bound capacity formulation, the notion of Most Likely To Fail (MLTF) element is introduced. MLTF element is defined as the member with the lowest capacity over stiffness ratio. The lower-bound lateral capacity of a jacket bay is estimated by adding the horizontal force components of all load carrying members in the given bay at the instant of first member failure. A linear multi-spring model is used to relate the forces and displacements of diagonal braces within a bay. The axial force in the jacket legs due to lateral overturning moment is estimated at each bay and its batter component is added to the lateral capacity.

An upper-bound capacity is also formulated for each bay. After the MLTF member (including its joint) in compression reaches its axial capacity, it can not maintain the peak load and any further increase in lateral displacement will result in unloading of this member. Presuming that the load path remains intact (inter-connecting horizontals do not fail), a load redistribution follows and other members carry the loading of the lost members until the last brace reaches its peak capacity.

An empirical residual capacity modification factor, α, is introduced (α = residual load capacity / peak load capacity). Assuming elasto-perfectly plastic material behavior, α is equal to 1.0 for members in tension (neglecting strain hardening effects) and less than 1.0 for members in compression due to P-Δ effects (generally, in the range of 0.15 to 0.50). The upper-bound capacity of a given jacket bay is estimated by adding the horizontal component of the residual strength of all of the braces within the bay.

Within the framework of a simplified analysis, the jacket has been treated as a trusswork. Plastic hinge formation in the jacket legs is not considered because this hinge development occurs at a lateral deformation that is much greater than is required to mobilize the axial capacities of the vertical diagonal braces. At the large lateral deformations required to mobilize the lateral shear capacities of the legs, the diagonal brace load capacities have decreased markedly due to column buckling or tensile rupture.

In general, the effect of bending moment along the jacket legs on the lateral capacity is neglected. This leads to estimates of lateral capacity that are either conservative or unconservative depending on the actual bending moment distribution in the legs. However, the difference in capacities (estimated vs. actual) is negligible for all but the uppermost and lowest jacket bays. Due to frame action in the deck portal and rotational restraint of the legs at mud level, the jacket legs experience relatively large bending moments at these two bays. The bending moment in the legs at the lowest bay has the direction of a resisting moment and hence not considering it can only be conservative. In contrary, the shear force due to the large moment gradient at the uppermost jacket bay has the same direction as the global lateral loading and hence reduces the lateral capacity. If this effect is not taken into account, the lateral capacity will be over-estimated.

A simplified procedure is developed to account for the effect of shear force in the top jacket bay. We are interested in moment distribution along the legs at this bay due to frame action in the deck portal. Given the geometry of the deck portal and the load acting on deck areas, the moment distribution along the deck legs is estimated. Thinking of a jacket leg as a continuous beam which is supported by horizontal framing, the applied moment at the top of the leg rapidly decreases towards the bottom. Based on geometry of the structure, in particular jacket bay heights and the cross-sectional properties of the jacket leg (if non-prismatic), and in the limiting case of rigid supports, an upper-bound for the desired moment distribution is estimated.

The braces are treated as though there are no net hydrostatic pressures (e.g. flooded members). Based on a three-hinge failure mode, the exact solution of the second order differential equation for the bending moment of a beam-column is implemented to formulate the equilibrium at collapse.

Elasto-perfectly plastic material behavior is assumed. The ultimate compression capacity is reached when full plastification of the cross-sections at the member ends and mid-span occurs. It is further assumed that plastic hinges at member ends form first followed by plastic hinge formation at mid-span.

The results have been verified with results from the nonlinear finite element program USFOS.[13,14] Using the same initial out-of-straightnes for both simplified and complex analyses, the axial compression capacity of several critical diagonal members of different structures has been estimated. The simplified method slightly over-predicts the axial capacity of compression members (less than 10%).

Given the conservative formulation of buckling capacities when compared with test data (refer to Commentary D in API RP 2A-LRFD guidelines)[9], this over-prediction may in fact be closer to the expected or best estimate capacity.

In case of dent damaged braces or braces with global bending damage, the axial capacity is reduced according to the equations given by Loh[15] which were developed for evaluating the residual strength of dented tubular members. The unity check equations have been calibrated to the lower bound of all existing test data. The equations cover axial compression and tension loading, in combination with multi-directional bending with respect to dent orientation.

Tubular Joint Capacity

The stress analysis of the circular tubular joints and the theoretical prediction of their ultimate strength has proven to be difficult. Hence, empirical capacity equations based on test results have often been used to predict the joint ultimate strength. For simple tubular joints with no gussets, diaphragms, or stiffeners, the capacity equations given in the API RP 2A LRFD guidelines are used (1993).

It is generally recognized that the equations for joint capacity are conservative. Bias factors (true capacity / nominal or guideline capacity) are provided in ULSLEA so that the user can utilize the expected or best estimate capacities of the elements to determine the capacity of the platform components (deck legs, jacket, foundation).

Pile Capacity

The pile shear capacity is based on an analysis similar to that of deck legs with the exception that the lateral support provided by the foundation soils and the batter shear component of the piles are included. Virtual work based limit equilibrium equations have been developed to characteize the ultimate limit state lateral loading capacity of piles embedded in cohesive and cohesionless soils.

The horizontal batter component of the pile top axial loading is added to estimate the total lateral shear capacity of the piles. This component is computed based on axial loads carried by the piles due to storm force overturning moment.

The axial resistance capacity of a pile is based on the combined effects of a shear yield force acting on the lateral surface of the pile and a normal yield force acting over the entire base end of the pile.

It is assumed that the pile is rigid and that shaft friction and end bearing forces are activated simultaneously. Correction factors can be introduced to recognize the effects of the pile shaft flexibility.

It is further assumed that the spacing of the piles is sufficiently great so that there is no interaction between the piles (spacing to diameter ratios exceed approximately 3). In the case of compressive loading, the weight of the pile and the soil plug (for open-end piles) is deducted from the ultimate compressive loading capacity of the pile. For open-end piles, the end bearing capacity is assumed to be fully activated only when the shaft frictional capacity of the internal soil plug exceeds the full end bearing.

PLATFORM VERIFICATIONS

In this paper we summarize results from five second generation analysis and verification studies of Gulf of Mexico template-type platforms. The verification cases include two eight-leg and one four-leg drilling and production platforms, and two, four-leg well protectors. These structures are identified as platforms 2A through 2E.

The simplified estimates of total forces acting on the platforms during intense storms and predictions of ultimate member strength and platform capacity developed using ULSLEA[5,6] were verified with results from nonlinear static, push-over analyses developed using the nonlinear finite element computer program USFOS.[14] Up to the first member failure, the USFOS analyses were load controlled. Thereafter, they were displacement controlled. Wave and wind loads in the deck were calculated and applied as nodal loads. The hydrodynamic forces on jacket were generated using the WAJAC wave load program.[16] Stokes 5th order wave theory was used and member loads were calculated based on the API RP 2A guidelines.[9,10]

None of the results reported herein have incorporated corrections to recognize wave dynamics - platform nonlinear response characteristics.[17] For the structures discussed in this paper, these corrections are indicated to increase the static lateral loading capacities by factors (Fv = dynamic lateral loading capacity / static loading capacity) in the range of Fv = 1.0 to 1.2.

The ULSLEA analyses were performed assuming elasto-perfectly plastic behavior for tubular brace members and joints in both tension and compression (a residual strength factor of a = 1.0) to estimate the maximum upper-bound capacities of jacket bays.[6] Values of a = 0.2 to 0.5 would be more representatives of most vertical diagonal braces.[5]

PLATFORM 2A

Platform 2A is an 8-leg structure located in the Main Pass area of the Gulf of Mexico in a water depth of 271 ft. Designed and installed in 1968-70, the platform has been exposed to high environmental loading developed by hurricanes passing through the Gulf. In 1979, hurricane Frederic passed nearby generating a maximum wave height of approximately 50 ft. (based on hindcast results and post-hurricane damage inspection results). The total lateral storm loadings were estimated to be approximately 2,000 kips. The platform sustained these loadings without significant structural damage.

The structure foundation consists of eight 42-in. piles which penetrate to a depth of 270 ft. into medium dense sands overlaying stiff to very stiff clays. The jacket legs are battered in two directions and the leg-pile annulus is grouted. The lower and upper decks are located at +46 ft. and +63 ft. respectively.

The ULS static push-over lateral loading capacities were determined for the platform's principal orthogonal directions. The results are summarized in Figure 1. In these analyses, both loading (in the elastic range) and displacement (in the plastic range) solution controls were used. In the case of end-on loading, the wave in deck condition resulted in an ultimate lateral load capacity of 2,700 kips. Most of the member failures were due to compressive buckling of braces. The analyses indicated a brittle strength behavior and little effective redundancy which is a typical result for K-braced platform systems. In the case of broadside loading with wave in the deck, the ultimate capacity was 2,900 kips.

The same oceanographic conditions and hydrodynamic coefficients utilized in the detailed analysis were used to perform a simplified analysis. For 100 year storm conditions, the simplified analysis indicated 3,400 kips and 2,900 kips total base shear for broadside and end-on loading, respectively (Figures 2 and 3).

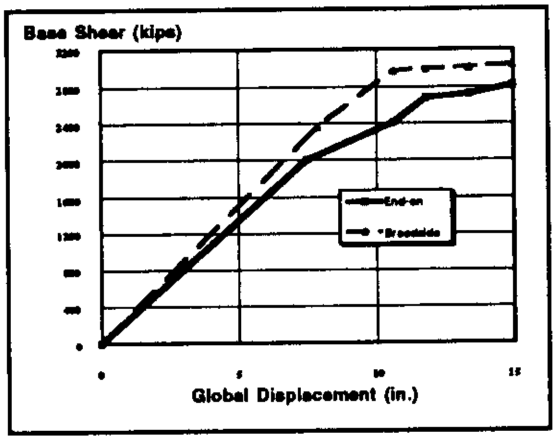


FIGURE 1: PLATFORM 2A BROADSIDE AND END-ON FORCE - DISPLACEMENT RELATIONSHIP

Compared with the results from detailed analysis, the total base shear is over-predicted by less than 15 %. The principal difference is due to modeling assumptions in the simplified analysis: all of the platform elements are modeled as equivalent vertical cylinders that are concentrated at a single vertical position in the wave crest.

The platform shear capacities and storm shears (abscissa) are plotted versus platform elevation (ordinate, above, +, below, -, mean sea level) in Figures 2 and 3. In broadside loading, ULSLEA predicted a failure mode in the second jacket bay at a total base shear of about 3,400 kips. In end-on loading, ULSLEA indicated a failure due to buckling of compression braces in the uppermost jacket bay at a lateral load of 2,900 kips (Figure 3).

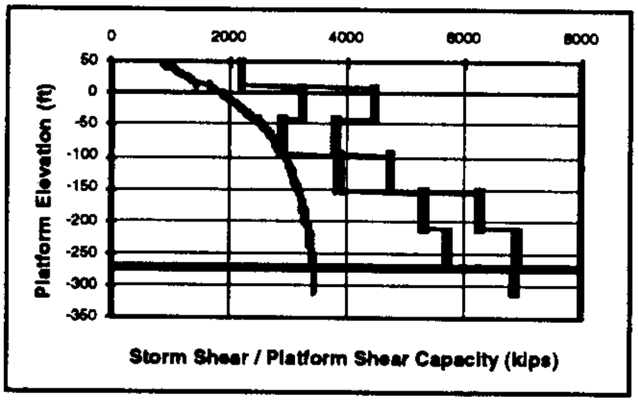


FIGURE 2: PLATFORM 2A BROADSIDE STORM SHEARS AND PLATFORM SHEAR CAPACITIES

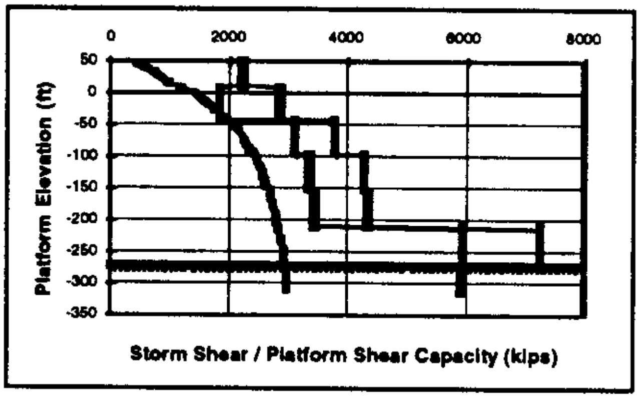


FIGURE 3: PLATFORM 2A END-ON STORM SHEARS AND PLATFORM SHEAR CAPACITIES

These results are 10 to 15% higher than those gained from detailed nonlinear analyses.[4] The principal difference lies in the nonlinear modeling of vertical diagonal braces which results in different buckling loads.[13]

Both the ULSLEA and detailed nonlinear analysis results are in conformance with the observed performance of the platform during hurricane Frederic. The platform survived this storm without significant damage and the results of the analyses indicate that it should have.

PLATFORM 2B

Platform 2B is an eight-leg structure located in a water depth of 118 ft.[18] The platform was designed using a design wave height of 55 ft. The cellar and main decks are located at +34 ft. and +47 ft., respectively. The 39 in. diameter jacket legs are battered in two directions and have no joint cans. The 36 in. diameter piles are grouted inside the jacket legs.

This platform sustained severe loadings from hurricanes Carmen (1974) and Andrew (1992).[18] The maximum wave height at the platform during hurricane Andrew was estimated to be 59 ft.[18, 19] The estimated maximum total lateral loading on the platform during hurricane Andrew was estimated to be approximately 3,700 kips. Damage sustained during Andrew indicated that the platform was loaded so that the upper bay of K-brace joints were loaded into the nonlinear range with two of the joints reaching their ultimate capacity.[18]

Nonlinear push-over analysis results summarized in Figure 4 indicated that the platform is capable of resisting approximately 3,900 kips in broadside loading.[4] The failure mechanism occurs in the uppermost jacket bay due to buckling of the compression braces and the associated joints. The analysis indicates a brittle strength behavior and little effective redundancy.

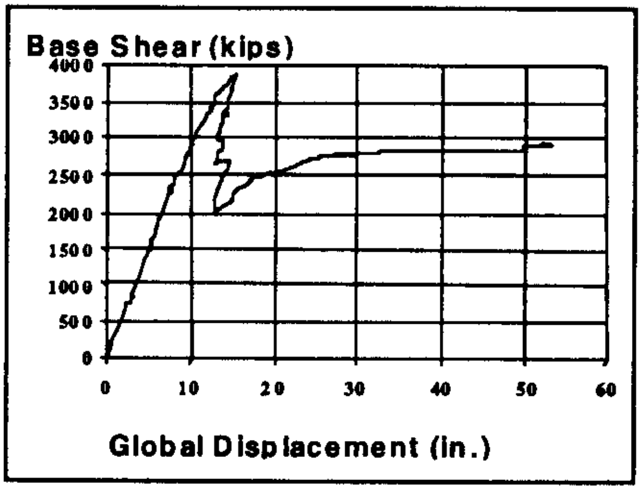


FIGURE 4: PLATFORM 2B BROADSIDE FORCE - DISPLACEMENT RELATIONSHIP

These results can be compared with those published by Imm, et al.[18] Their broadside static-push over analysis was based on an Andrew loading pattern that did not involve deck loadings. The static push-over analyses reported here did involve deck loadings.[4] The results reported by Imm et al.[18] indicated a total lateral loading capacity of approximately 4,900 kips. As noted by Imm, et al., the loading pattern used to perform the static push-over analyses can have a marked influence on the ultimate limit state performance of the structure. In this case, the lateral loading capacity involving deck loadings is 80 % of the lateral loading capacity without deck loadings.

The predicted lateral loading capacity and failure mode is in agreement with the observed platform performance in hurricane Andrew.

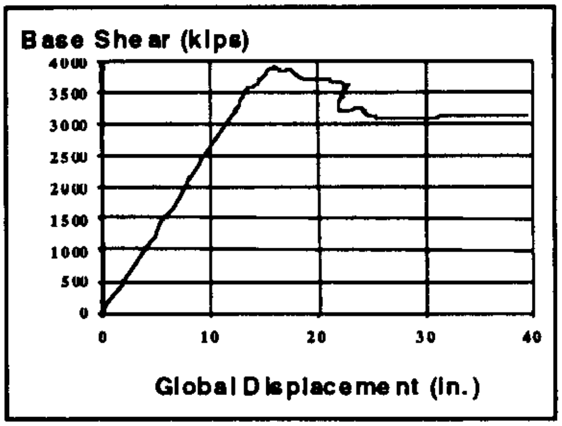


FIGURE 5: PLATFORM 2B END-ON FORCE - DISPLACEMENT RELATIONSHIP

The analysis showed the platform's end-on resistance capacity to be approximately 3,900 kips (Figure 5). Failure begins in the uppermost jacket bay, where the four diagonal compression braces and associated joints buckle almost simultaneously. The failure mechanism is completed when the horizontal struts in the upper jacket bay buckle in addition to compression braces and joints.[4]

The same oceanographic conditions, hydrodynamic coefficients, and wave theory utilized in the nonlinear push-over analyses were used to perform an ULSLEA. Since the same procedure was used to estimate the wind and wave forces on the projected deck areas, they were essentially the same for both detailed and simplified analyses. The resulting storm shears are summarized in Figures 6 and 7.

In broadside loading direction, the simplified force calculation procedures over-estimated the hydrodynamic loads on the jacket by 7 %. In end-on loading direction, the jacket loads were over-estimated by 15 %.

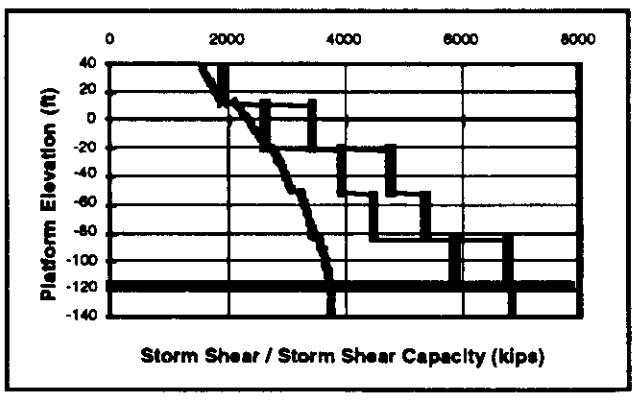


FIGURE 6: PLATFORM 2B BROADSIDE STORM SHEARS AND PLATFORM SHEAR CAPACITIES

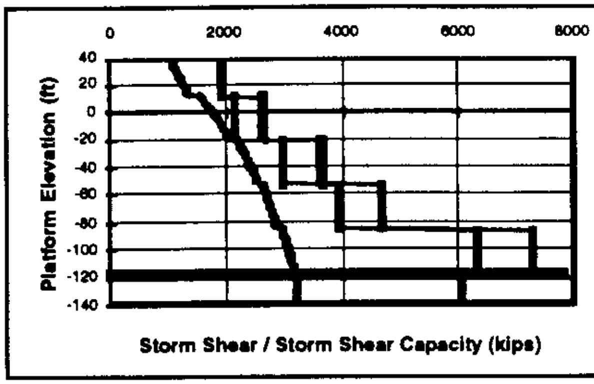


FIGURE 7: PLATFORM B END-ON STORM SHEARS AND PLATFORM SHEAR CAPACITIES

For each loading direction, the predicted performance of MLTF vertical diagonal brace has been verified. Using the same initial out-of-straightness for both simplified and complex analyses, the simplified column buckling formulation over-predicted the peak member load by 6 % and 9 % for end-on and broadside loading directions respectively. Using the calibrated format of simplified column buckling equations with a buckling length factor of K = 0.65, the simplified analysis under-predicted the peak load by 7 % and 1 % for end-on and broadside loading directions respectively.

To study the effect of K-factor on predicted buckling load, a sensitivity analysis was performed. The calibrated buckling capacity formulation gave the "exact" result when buckling length factors of K = 0.65 and 0.55 were used for MLTF members in compression for broadside and end-on loading directions, respectively. Note that in the latter case, the brace is connected to jacket legs at both ends and is therefore stiffer. It is interesting to note that this result is in good agreement with those presented by Hellan, et al.[13]

The platform shear capacity and storm shear profiles are plotted versus platform elevation in Figures 6 and 7. In the case of broadside loading and using a buckling length factor of K = 0.65 for braces in compression, ULSLEA predicted a failure mode in the deck legs and uppermost jacket bay at a total base shear of about 3,700 kips, which is in good agreement with the results from the non linear analysis (~ 6 % under-prediction). In case of end-on loading with a buckling length factor of K = 0.55 for compression braces, the simplified analysis predicts a collapse load of 3,100 kips (~ 20 % under-prediction) due to failure of compression braces and joints in the top jacket bay. The ULSLEA predictions are in accord with the observed behavior of this platform during hurricane Andrew.

PLATFORM 2C

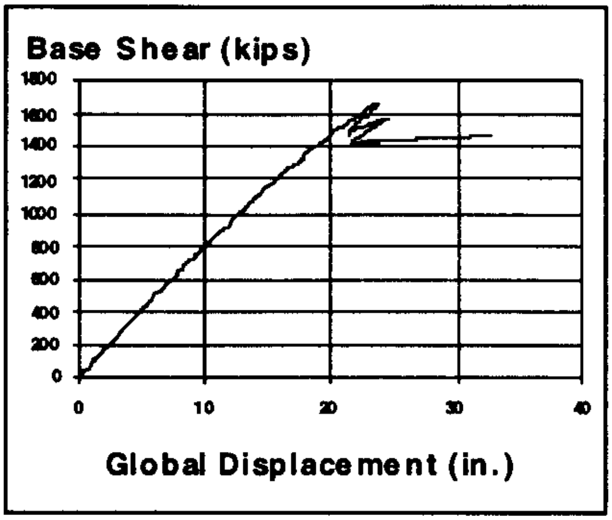
Platform 2C is a four pile drilling and production platform. It was installed in the Gulf of Mexico Ship Shoal region in a water depth of 157 ft. in 1971. The platform has four decks at elevations +33 ft., +43 ft., +56 ft., and +71 ft. The deck legs form a 30 ft. By 30 ft. Plan and the jacket legs are battered in two directions (1:11) and have joint cans. The leg-pile annulus is ungrouted and the 36-in. Diameter piles are attached to the jacket with welded shimmed connections at the top of the jacket. The vertical bracing is comprised of horizontal K-braces.[4]

The piles extend 355 ft. Below 28 ft. of soft to stiff gray clay and 27 ft. of fine dense sand. The sand layer starts at 197 ft. Below the mudline. The clay above the sand is generally soft and silty, while the clay below the sand is stiff to very stiff.

This platform was located close to the track of hurricane Andrew. The estimated wave height at the platform location was estimated to be approximately 60 ft. The platform survived the storm without significant damage.

This platform has been the subject of extensive structural analyses.[20] As part of an industry wide effort to assess the variability in predicted performance of offshore platforms in extreme storms, the storm loadings and ultimate capacity of this "benchmark" platform has been assessed by 13 qualified investigators using a variety of nonlinear analysis software packages. All of the analysts were given the same platform drawings, soil conditions, and oceanographic conditions. It was specified that the storm loadings should be computed according to API guidelines.[9,10] It is noteworthy that the range of broadside lateral loading capacities was from 1,600 kips to 3,400 kips; a range in excess of 2 (mean value of 2,400 kips with Coefficient of Variation of 22 %).

Platform 2C was analyzed using USFOS.[14] As for all of the nonlinear analyses, an attempt was made to use "unbiased" characterizations for all loading and capacity factors to develop best estimate lateral loadings and capacities. The results from the USFOS static push-over analyses of platform 2C are summarized in Figures 8 and 9. These results indicated a maximum total lateral loading of 2,900 kips and a lateral capacity of 1,700 kips to 3,400 kips. This range brackets the range developed in the "benchmark" study.[20]



DISPLACEMENT RELATIONSHIP BASED ON STATIC PILE CHARACTERISTICS

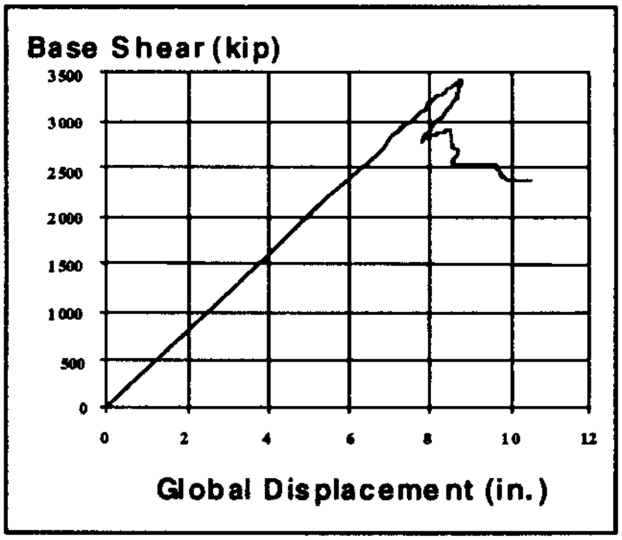


FIGURE 9: PLATFORM 2C FORCE - DISPLACEMENT RELATIONSHIP FOR DYNAMIC PILE CHARACTERIZATION

The range in lateral capacity was a function of how the foundation piles were modeled. If "static" capacities were utilized (based on the sampled soil strength test results and static pile capacity methods)[21], the initiating failure mode was in the foundation and the lower lateral loading capacity resulted. If "dynamic" capacities (based on corrected soil strength results to reflect the sampling disturbance and cyclic - dynamic loading effects) were utilized[21-23], the initiating failure mode was in the jacket and the upper lateral loading capacity resulted. As found in previous analyses[7,8], the methods used to evaluate and model the performance characteristics of the pile foundations can have marked effects on the platform lateral loading capacity.

It is extremely important to recognize the potential conservative biases introduced by soil sampling, testing, and pile capacity analysis procedures.[21, 24, 25] For these soils, the biases in pile axial capacity (ratio of expected capacity to predicted static capacity) can easily be in the range of 2 to 3.[21,24]

Using the simplified approach for a reference wave height of 67 ft., a wave period of 14.3 sec and a uniform current velocity of 3.1 ft. per sec., the total base shear for an orthogonal loading direction was estimated to be 3,100 kips (Figure 10). Using a buckling length factor of 0.65 for compression braces, ULSLEA indicated platform collapse at a base shear of 3,200 kips due to simultaneous failure of compression braces at three different jacket bays (Figure 10). For this lateral loading, the mean axial pile static capacity in compression was exceeded by approximately 30 % (RSR = 0.7). According to this "best estimate" result, a failure mode in foundation would govern the ultimate capacity of the platform. However, recognition of dynamic loading effects in the foundation indicated that the failure mode would be in the jacket rather than in the pile foundation.

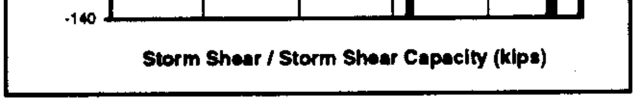


FIGURE 10: PLATFORM 2C STORM SHEARS AND PLATFORM SHEAR CAPACITIES (DYNAMIC FOUNDATION CONDITION)

These results are in good agreement with those gained from detailed nonlinear analyses.[4] The comparison indicated that the simplified method over-estimated the current and wave loads in jacket by 17 %. The ultimate capacity of the platform with the dynamic pile foundation characteristics was under-predicted by 6 %. The axial compression capacity of piles were over-estimated by 14 %. After including the self-weight of the jacket in the axial pile loading, the pile capacities were in close agreement. Due to how the piles are installed and the potential loadings carried by the mudline braces and mudmats, whether or not the dead loads are fully carried by the supporting piles is uncertain.

PLATFORMS 2D AND 2E

Platforms 2D and 2E are four pile well protectors, both located in the South Timbalier area. The two wellhead protectors were designed and installed early in the 1980's by the same firm. The two wellhead protectors were designed according to the same API RP 2A guideline. The slightly older platform 2D is located in 52 ft. of water and is oriented 45° counterclockwise from true north. Platform 2E is located in slightly shallower water (49 ft.) and is oriented parallel to true north. Both structures are two bay, nearly symmetrical, four pile template structures designed to provide limited facilities for 36 in. diameter caisson well risers. Both protectors have offset braced helipads.

The jacket framing of the two structures is almost identical, with platform 2E having slightly smaller diameter jacket legs and piles; 28 and 24 in., as opposed to 30 and 26 in. Diagonal vertical K-bracing is made up of 18 in. tubulars, while plan bracing is composed of 12.75 in. tubulars on all three levels. All members were fabricated using A36 grade steel.

The most prominent difference between the two structures, other than water depth and orientation, lies in the number and location of caisson risers each structure must support. The two caissons of platform 2D are located just outside of the structure north end of the jacket and are not tied substantially to the jacket. The caisson in platform 2E is rigidly framed within the interior of the jacket.

The foundations for the two structures are very similar only in that they are both composed of four piles. The design of these piles is quite different. Platform 2D's piles are 187 ft. long. 26 in. in diameter and are comprised of several segments. Platform 2E's piles are slightly longer (190 ft.) than those of Platform 2D to compensate for its smaller diameter of 24 in. It's upper wall thickness are generally larger as well, running at 1.213 in. to withstand the large bending stresses found in the piles near the mudline. The remaining distribution is essentially the same as for Platform 2D.

During hurricane Andrew, Platform 2D collapsed (tension piling pull-out and brace failures) and Platform 2E survived without significant damage.

The analyses of Platforms 2D and 2E were performed in stages, progressively using more complex analyses ranging from ULSLEA to StruCad*3D[26] and to USFOS[14]. During the initial analyses, the predicted behavior of the platforms was not in conformance with the observed performance during hurricane Andrew. This motivated a detailed study of the platform construction and installation records. During this study, it was discovered that the piling on the south side of Platform 2D had been under-driven by 5 to 10 ft. This finding was integrated into the analyses reported here. This experience pointed out the importance of having very detailed information on platforms that are loaded close to their ultimate limit states. Without such information, observations of failures and non-failures might be attributed to "probabilistic reasons"[27] when the real reasons are founded in deterministic characteristics.

The two structures were loaded only along their principal axes to provide consistency between the various approaches employed to analyze structural response. Wave loads for USFOS were generated by the program WAJAC.[16] The global base shears developed on Platform 2D and Platform 2E during the passage of Andrew were based on hindcast study results.[19] The results indicated Platform 2D experienced peak lateral loadings that were about 20 % larger than those on Platform 2E. During hurricane Andrew, the hindcast peak lateral loading on Platform 2D was 1,100 kips and on Platform 2E was 850 kips.[4]

The static push-over results for Platform 2D and Platform 2E based on the USFOS results are summarized in Figure 11. The "double humps" in the load - displacement results are due to the increased stiffness of the structures when contact between the jacket and caissons occur. The negative stiffness found at the end of all analyses represents pile pullout. The large lateral deformations produce plastic hinges in the piles which produce a near mechanism. It is the additional strength and rigidity of the caissons which prevents the structures from soft story collapse. This added stiffness allows the full axial capacity of the soils to be exceeded to produce pile pullout.

The USFOS results indicated that the maximum lateral load capacity of Platform 2D (end-on and broadside loadings) is 910 kips and Platform E 880 kips.

The USFOS results indicated that the ratio of the peak lateral loading during hurricane Andrew to the maximum lateral loading capacity is 1.2 and 0.95 for Platform 2D and Platform 2E, respectively. The analyses indicate that Platform D should have failed due to pile pullout and Platform E should have survived.

The paradox of why these two seemingly identical structures behaved differently was due to the differences in the appurtenances (well conductors), the manner in which the wells were tied into the structures, and the under-driven piles in Platform 2D. The effects of these differences only became evident when these "details" were determined and their implications integrated into the analyses. The results from the analyses were in conformance with the observed behavior of the platforms.

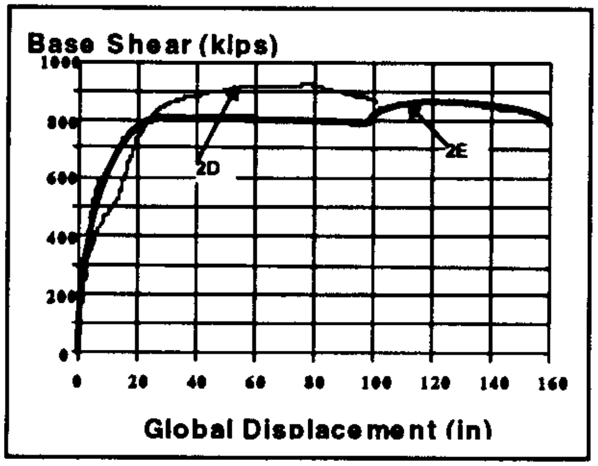


FIGURE 11: PLATFORMS 2D & 2E LOAD - DISPLACEMENT CHARACTERISTICS

Figures 12 and 13 summarize the ULSLEA analysis results (end-on results shown, broad-side results were comparable). The results indicate that the lateral loading capacity of Platforms 2D and 2E would both be about 1,100 kips. The ULSLEA results indicated that the maximum lateral load capacity of the two platforms was about 1,100 kips, resulting in an overestimated capacity of 21 % and 25 %, respectively.

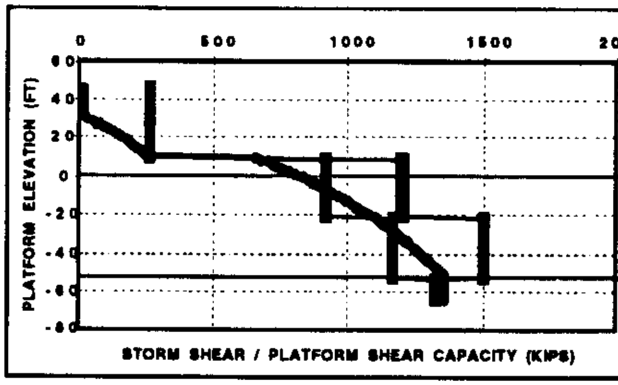


FIGURE 12: PLATFORM 2D STORM SHEARS AND PLATFORM SHEAR CAPACITIES

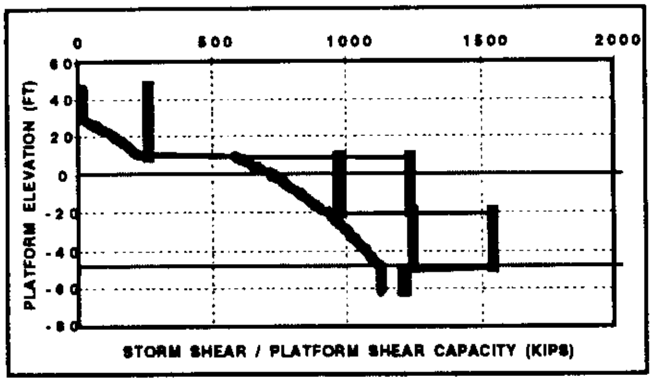


FIGURE 13: PLATFORM 2E STORM SHEARS AND PLATFORM SHEAR CAPACITIES

The simplified analyses indicate that the failure mode in Platform 2D would involve the pile pullout and failure of the vertical diagonal braces. Given the computed lateral loadings during hurricane Andrew, the ULSLEA analyses would indicate that Platform 2D could be expected to fail and Platform 2E could be expected to survive. The ULSLEA analysis results were in conformance with the observed performance of the platforms during hurricane Andrew.

SUMMARY AND CONCLUSIONS

Simplified procedures have been developed that will permit evaluation of the environmental loadings and structural performance of template-type platforms under extreme storm conditions. The results summarized in Table 1 and those previously published[7,8,28] indicate that ULSLEA can develop evaluations of both storm loadings on and ultimate lateral capacities of platforms that are excellent approximations of those derived from complex analyses. The verifications of the second generation procedures embodied in ULSLEA indicate biases (USFOS capacity / ULSLEA capacity) in the range of 0.80 to 1.22 with a mean value of 0.95. These results are comparable with those developed during the verifications of the first generation simplified procedures.[7,8] The ULSLEA procedures seem to produce more consistent results than the "equivalent linear ULS" approach advanced by Vannan, et al.[29]

Comparison of the estimated lateral load capacities with the estimated maximum loadings that these platforms have experienced and with observed performance characteristics of these platforms indicates that the analytical evaluations of both storm loadings and platform capacities are also in good agreement with the experience.

The use of the simplified analytical procedures to estimate reference storm lateral loading and platform capacities, and Reserve Strength Ratios are indicated to result in good estimates that can be used in the process of screening platforms that are being evaluated for extended service.[28] In addition, the results from these analyses can be used to help verify results from complex analytical models that are intended to determine the ultimate limit state loading capacities of platforms. Lastly, this approach can be applied as a preliminary design tool for configuration of new platforms.

CONTINUING WORK

This study is part of a multi-year joint industry - government sponsored research project to develop simplified methods to analyze the static and dynamic ultimate limit state performance characteristics of platforms. At the present time, detailed nonlinear analyses are being performed on two additional 8-leg platforms that were subjected to storm loadings by hurricane Andrew.[29] One of these seemingly identical 8-leg platforms failed and the other did not. Two other 8-leg platforms are also being studied. These platforms survived loadings developed by hurricanes Hilda and Camille.[7,8] Verification of ULSLEA with these results will be the subject of future publications.

ULSLEA has a probability component that allows the user to define loading and capacity uncertainties and biases and determine a probability of failure of the platform.[7,8] This component has been verified with results from sophisticated structural system reliability analyses. The background on this development and verification will be the subject of future publications.

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TABLE 1: COMPARISON OF USFOS AND ULSLEA RESULTS

Platform	Configuration	Wave Direction	ULSLEA Collapse Base Shear (kips)	USFOS Collapse Base Shear (kips)	Ratio of USFOS / ULSLEA Base Shears
2A	8 leg double battered K-braced	End-on Broadside	2,900 3,400	2,600 2,900	0.90 0.85
2B	8 leg double battered K-braced	End-on Broadside	3,100 3,700	3,900 3,900	1.22 1.05
2C	4 leg double battered horizontal K-braced	End-on (dynamic) End-on (static)	3,200 2,000 (1,700)*	3,400 1,700	1.06 0.85 (1.00)*
2D	4-leg double battered vertical K-braced	End-on	1,100	910	0.83
2E	4-leg double battered K-braced	End-on	1,100	880	0.80

* includes platform deadweight in pile axial loading

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